FIELD STUDIES OF POST-TENSIONED BRIDGE HINGE CURL

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ABSTRACT

Accurate prediction of instantaneous and time dependent deformation of superstructure in-span hinges is important to avoid mismatch at the intermediate expansion joints of bridges. When the two sides of an in-span hinge are at different elevations, road hazard is created leading to possible accidents or damage and excessive wear and tear to vehicles. Hinge curl refers to deformation of the superstructure at the hinge caused by post tensioning forces. A commonly used method to estimate hinge curl was developed by the California Department of Transportation (Caltrans) through Memo to Designers (MTD) No. 11-34. However, this method does not always lead to accurate estimate of deformations and, hence, grinding of the superstructure at the hinge is often necessary. The principal aim of the study is to evaluate Caltrans equations used for calculating "hinge curl" deflections and propose new methods to more accurately estimate hinge curl effect. Field data are being collected from five bridges in California after completion of superstructure but prior to post-tensioning, immediately after post-tensioning, and then approximately once a month until the bridge is opened to traffic. Field data obtained for these bridges immediately after stressing indicate that measured hinge curls substantially exceed those obtained from current design equations. The data have revealed the primary sources of discrepancy and have provided a direction to take corrective measures.

Keywords: Post-tensioning, Hinge Curl, Assessment and Monitoring, Research.

INTRODUCTION

For cast-in-place (CIP) post-tensioned concrete (PS) box girder bridges, the in-span hinge as shown in Fig. 1 is an important element that requires special consideration with respect to design, detailing and construction sequencing. In-span hinges are used in the superstructure of long bridges to divide the structure into shorter frames to reduce the stresses in the columns resulting from temperature, creep and shrinkage forces. The upward deflection caused by post-tensioning forces of the short cantilever of in-span-hinges is termed as "hinge-curl". Inaccurate estimation of hinge curl can lead to unsatisfactory service conditions due to excessive deformations at the hinge. Discrepancy between the elevations of the two sides of the hinge creates a bump on the road, which could be a road hazard. To have equal road profile at both sides of the hinge, adjustment in the long cantilever falsework or concrete grinding at the hinge is required (Fig. 2). This results in extra cost and delay, and could reduce the concrete cover on the deck reinforcement making the deck steel susceptible to corrosion.

Fig. 1 In-span hinges in CIP/PS box girder bridges

Fig. 2 Superstructure concrete grinding and demolishing at an in-span hinge (Bradley Overhead Replace)

Deformation prediction in prestressed concrete bridges is marked with relative uncertainty, mainly due to time dependent concrete creep, and prestress losses. Many studies have been conducted on the concrete creep behavior as well as the prestress losses. Several empirical equations have been proposed for creep coefficient and prestress losses[1,2,3]. The empirical equations were usually calibrated with testing small specimens, and measured data from real prestressed concrete bridges over long period are very limited. A field study carried out on a simply supported post-tensioned box girder bridge in Nevada showed difference between the measured and predicted mid-span deflection values because boundary conditions of the bridge were not modeled accurately in the routine calculation[4].

There is no study in the literature on hinge curl of post-tensioned concrete bridges. California Department of Transportation (Caltrans) proposed a method to predict the hinge curl (Memo to Designers (MTD) No. 11-34[5]. Although this method is being used by bridge designers, significant discrepancy between the calculated and the measured hinge curl immediately after the post-tensioning has been reported by field engineers.

The study presented in this paper was conducted to investigate the correlation between the actual hinge curls occur immediately after post-tensioning and those estimated using MTD 11-34 equations.

FIELD MEASURMENTS AND DISCRIPTION OF THE BRIDGES

Vertical deflection of the short cantilevers at in-span hinges was measured as primary database in this study. In addition to the deflection, temperature and relative humidity were measured in the field. The data collection started about one week after the concrete casting of the superstructure. Two data sets were collected for this study. One data set was collected prior to the post-tensioning as a reference data and one immediately after post-tensioning. Five bridges across the State of California are included in the hinge curl research project. The hinge curl monitoring has been completed for three bridges so far and is in progress for the other two bridges. In this article the first three bridges are discussed.

The post-tensioning force is usually applied 10 days after casting the deck slab as shown in Fig.3. The superstructure concrete strength should gain at least 70% of its concrete compressive strength at 28 days at time of stressing.

Fig. 3 Post-tensioning process at hinge 7 (San Luis Rey River Bridge)

However given the fact that the soffit and stems are cast earlier than the deck and from the collected information about the tested concrete field samples, the box girder concrete for all three bridges had the specified 28-day compressive strength, f'_c by the time of stressing.

Bridge movements are measured on top of the superstructure. A station gridline is marked on the deck surface in the longitudinal and transverse direction of the bridge. Number of longitudinal gridlines depends on the bridge width.

Figures 4, 5 and 6 show the bridge components terminology used for this study.

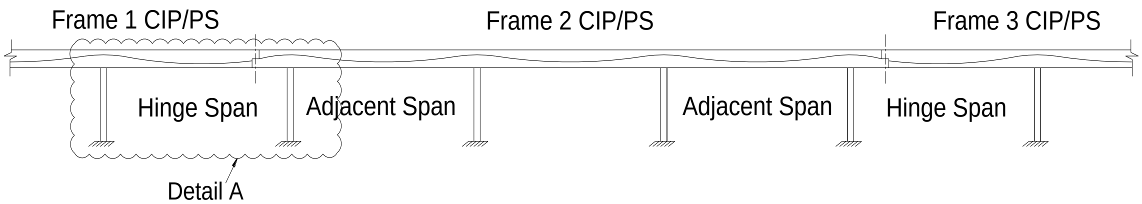
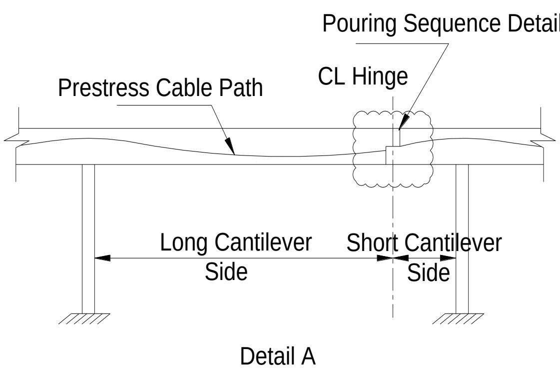


Fig. 4 Schematic view of typical bridge with two in-span hinges



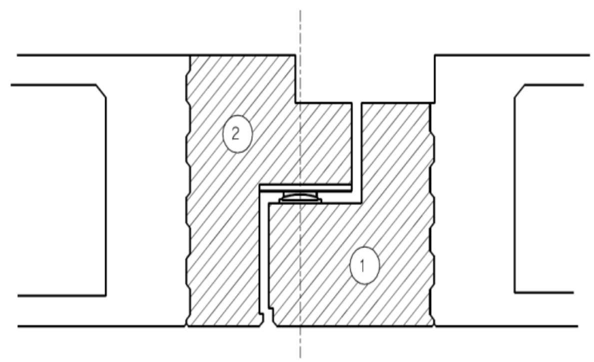


Fig. 5 Sketch of typical hinge span

Fig. 6 Hinge closure pouring sequence

Table 1.Summary of bridges

Bridge number	1		2		3
Bridge name	SAN LUIS REY RIVER BRIDGE		N170-N5 CONNECTOR		BRADLEY OVERHEAD (REPLACE)
Bridge no.	57-1208R		53-2976		39-0044
Year of construction	2011		2012		2012
Hinge label	3	7	1	2	Hinge
Jacking force (kips)	14754		15910		8745
Box Girder Width/Depth (ft)	58/7.9	46.3/7.9	58/9		30/8
Number of girders	5		4		3

General information for the bridges used in this study is listed in Table 1. Bridge 1 and 2 have three CIP/PS frames, and frame 2, where the short cantilevers exist, is the focus of the study. Bridge 3 has two CIP/PS frames and frame 1 is the surveyed one where the short cantilever exists. Two-end stressing was performed for frames 2 in bridges 1 and 2, and frame 1 in bridge 3. The stressing ends were at the two hinges for bridges 1 and 2, while they were at the hinge and the abutment in bridge 3.

DATA ACQUISTION AND DEFLECTION MEASURING METHOD

The vertical deflection of the short cantilevers and adjacent spans was measured using digital surveying level with a sensitivity of 1 mm (Fig. 7). During each data set measurement, ambient temperature and relative humidity were recorded using LCD Digital Thermometer Hygrometer placed in shade (Fig. 8).

The measurement stations were marked on the superstructure deck over the short cantilevers as shown in Fig. 9 using marking spray paint. A sketch of measurement stations gridline used for bridge 2 is shown in Fig. 10. For bridge 3, similar pattern of gridlines was implemented but with only two longitudinal gridlines A and B as bridge 3 has smaller width. For bridge 1, gridlines marked by Caltrans crew were used, which consisted of two transverse gridlines on the short cantilever at unequal distances because other desired stations were not accessible.

Fig. 7 Digital surveying level and rod

Fig. 8 Thermometer Hygrometer

Fig. 9 Measurement station marked on

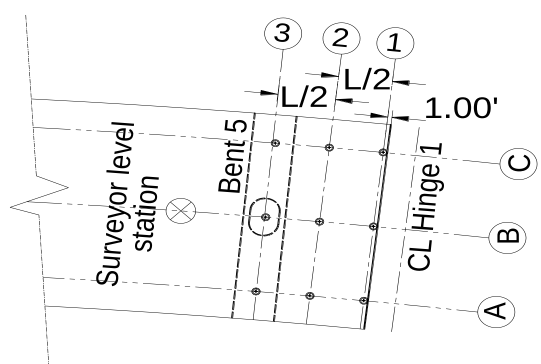


Fig. 10 Typical sketch of measurement

superstructure deck stations gridline (N170-N5 Connector)

FIELD MEASURMENT OBSERVATION

The measured deformations of the short cantilevers immediately after stressing in the three bridges are shown in Figs. 11 to 20. Relative movement of these cantilevers was measured with respect to the cap beam near the hinge. The hinge curl at the hinge centerline was extrapolated from the deformed shape of the short cantilever because the hinge centerline is at the closure pour. Placing the hinge closure may take several days after post-tensioning is completed. Deformed shapes along the longitudinal gridlines for each bridge were averaged in the transverse direction at each hinge.

The observed deformation behavior of these bridges appears to have similar and consistent trend. The measured values are different from bridge to bridge because of the variation in each bridge characteristics in terms of the dimensions, concrete properties, prestressing forces, and tendon eccentricities.

Minor difference in hinge curls at hinges 3 and 1 can be observed in bridges 1 and 2 as shown in Figs. 11 and 15 respectively. This difference could be attributed to the disparity of the post-tensioning forces in the bridge girders resulted from the tensioning sequence. The tensioning sequence varies the elastic shortening losses in the tendons and leads to asymmetrical forces on the bridge section. The difference in hinge curls is not pronounced at other hinges as shown in Figs. 13, 17 and 19. These hinges were the second stressing ends in those bridges, and accordingly variations in the elastic shortening losses were minimized. Hence the post-tensioning forces in the bridge girders were nearly symmetrical.

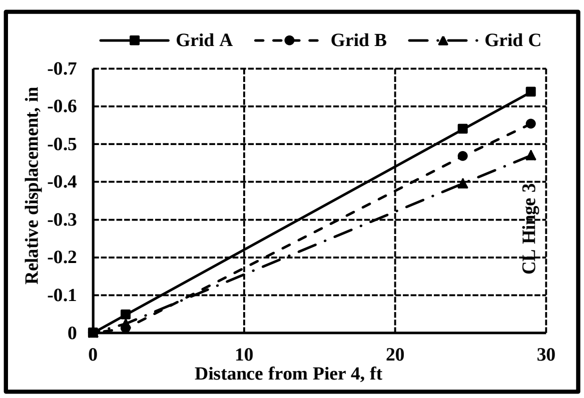


Fig. 11 Instantaneous hinge curl at hinge 3, bridge 1

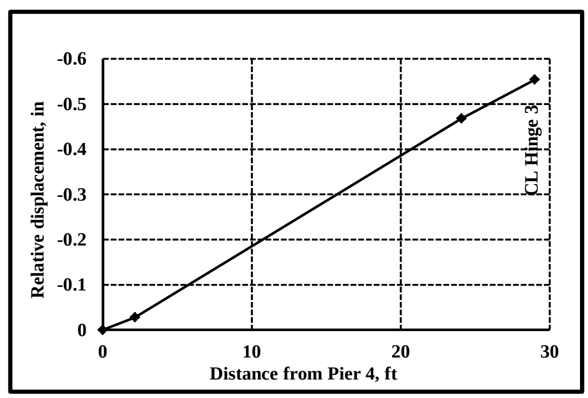


Fig. 12 Average hinge curl at hinge 3, bridge 1

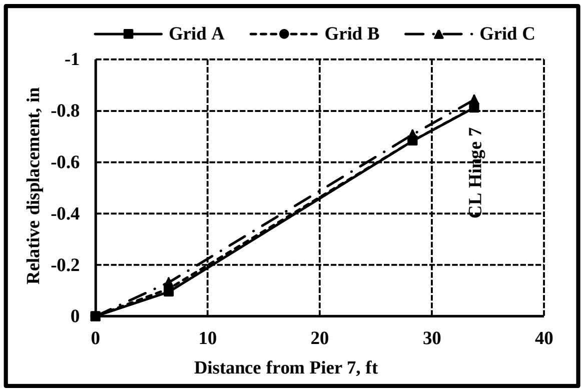


Fig. 13 Instantaneous hinge curl at hinge 7, bridge 1

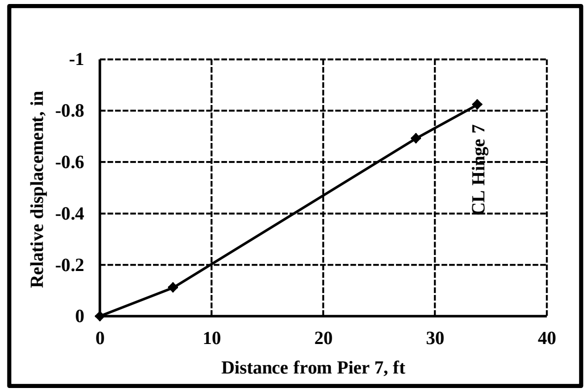


Fig. 14 Average hinge curl at hinge 7, bridge 1

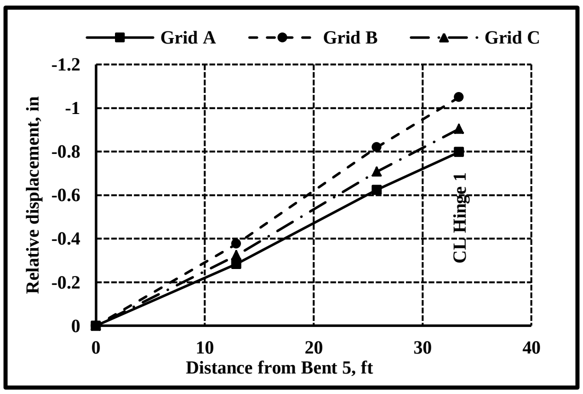


Fig. 15 Instantaneous hinge curl at hinge 1, bridge 2

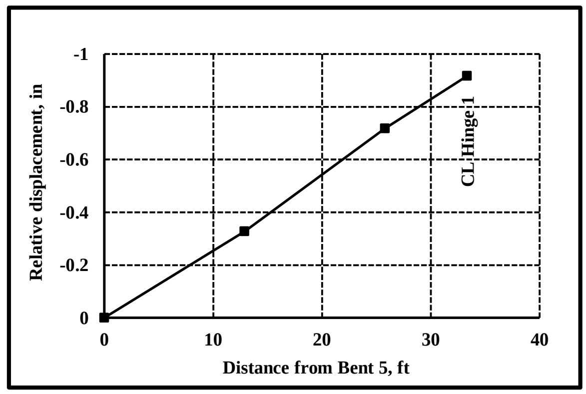


Fig. 16 Average hinge curl at hinge 1, bridge 2

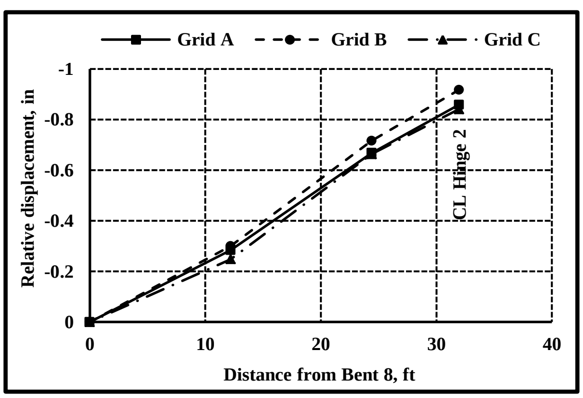


Fig. 17 Instantaneous hinge curl at hinge 2, bridge 2

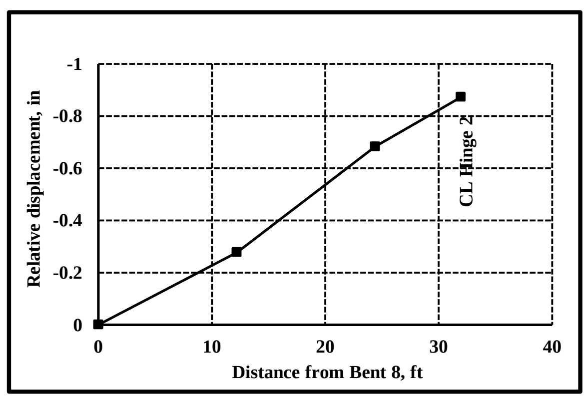


Fig. 18 Average hinge curl at hinge 2, bridge 2

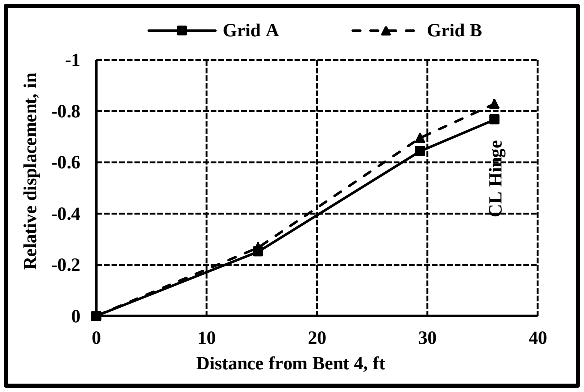


Fig. 19 Instantaneous hinge curl at hinge, bridge 3

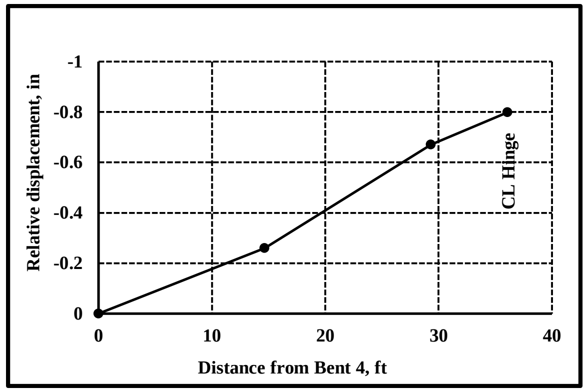


Fig. 20 Average hinge curl at hinge, bridge 3

CALTRANS METHOD (MTD 11-34) FOR HINGE CURL PREDICTION

Caltrans method for hinge curl prediction (MTD 11-34)[5] utilizes deflection factors instead of considering the time dependent changes in concrete modulus of elasticity, creep, shrinkage, and steel relaxation.

The long term effects are lumped and applied by deflection factor (Fig. 23) in MTD 11-34. The deflection factor curve represents the total amount of deflection of a cast-in-place prestressed concrete element with respect to time. The long term effect of creep and shrinkage is assumed to result in a total deflection that is three times that of immediate elastic deflection and will occur over a four year period. Although Fig. 26 shows up to 360 days, the deflection factor curve approaches 3.0 by day 1440. The MTD 11-34 method is summarized in the following steps:

1- Approximate the deflection of the short cantilever at the centerline of hinge due to dead load.

$$\Delta_{DL} = \underbrace{\frac{w.L_1^{\,3}}{24EI}\left(4L_3 - L_1\right)}_{(a)} + \underbrace{\frac{P.L_2^{\,2}}{6EI}\left(3L_3 - L_2\right)}_{(b)} \tag{1}$$

Where,

(a) Deflection of short cantilever due to self-weight (in)

(b) Deflection of short cantilever due to weight of short cantilever portion of hinge diaphragm (in)

w = Uniform self-weight of the prismatic section of the short cantilever (kips/in)

P = Weight of the portion of the hinge diaphragm that fills the voids of the prismatic section; short cantilever side only (kips)

L_1 = Length of short cantilever measured from the face of the hinge diaphragm to the face of support (in), (Fig. 21).

L_2 = Length of short cantilever measured from the face of support to the centroid of the short cantilever hinge diaphragm (in), (Fig. 21).

L_3 = Length of short cantilever measured from the face of support to the centerline of the hinge (in), (Fig. 21).

E = Concrete modulus of elasticity based on f'_c (ksi)

I = Average moment of inertia of short cantilever span (in^4)

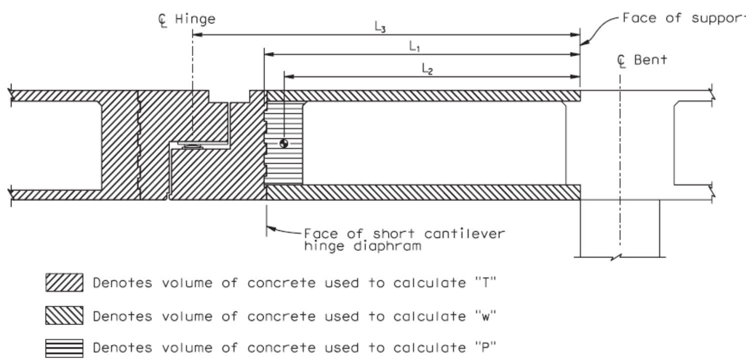


Fig. 21 Dead load for short cantilever

2- Approximate the deflection of the short cantilever at centerline of hinge due to prestress force.

$$\Delta_{PS} = \frac{-P_j . FC . L_1}{12EI} . \; e_1 \; 8L_3 - 3L_1 \; + e_2 \; 4L_3 - 3L_1 \qquad (2)$$

Where,

P_j =Design jacking force (kips)

FC =Average initial force coefficient at time of stressing in the short cantilever (unitless)

e_1 =Eccentricity at centerline of bent, positive up (in), (Fig. 22).

e_2 =Eccentricity at anchorage in hinge diaphragm, positive up (in), (Fig. 22).

3- The hinge curl

$$\Delta_{curl} = \Delta_{DL} + \Delta_{PS} \qquad (3)$$

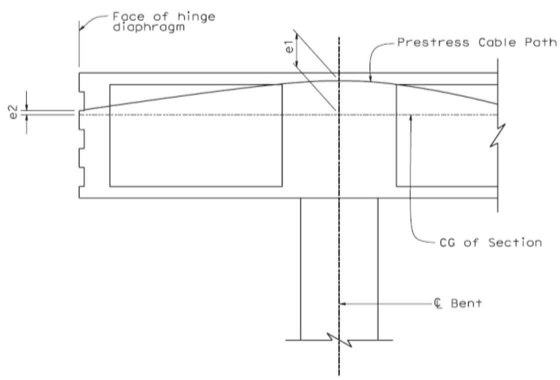


Fig. 22 Prestress cable path

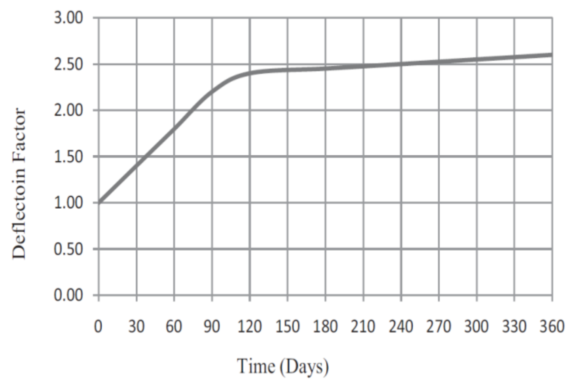


Fig. 23 Deflection factor chart

ANALYSIS OF RESULTS

Comparison between the measured average hinge curls immediately after the post-tensioning for bridge 1, 2, and 3 and those estimated using MTD 11-34 is listed in Table 2. To account for the duration over which stressing process is performed, which has taken on average two days to complete two end stressing for each frame in the three bridges, a deflection factor of 0.013 accounts for creep deformations over two days was applied to the instantaneous estimated hinge curl using MTD 11-34.

Table 2.Comparison of measured to estimated hinge curl

		Measured Δ_{curl} (inch)	Estimated Δ_{curl} (inch)	1.013* Δ_{curl} (inch)	Absolute difference (inch)	Difference %
Bridge 1	Hinge 3	0.555	0.112	0.113	0.442	391
	Hinge 7	0.824	0.215	0.218	0.606	278
Bridge 2	Hinge 1	0.917	0.128	0.130	0.787	605
	Hinge 2	0.872	0.117	0.119	0.753	633
Bridge 3	Hinge	0.798	0.246	0.249	0.549	220

It can be seen from Table 2 that the variation of the bridges dimensions and prestressing details led to different measured hinge curls. The comparison listed in Table 2 showed that MTD 11-34 substantially underestimates the hinge curl because the measured hinge curls were considerably higher than those estimated.

The cap beam rotation at the bent near the hinge due to falsework flexibility under the adjacent span is believed to be the primary reason for this large discrepancy between the measured hinge curls and those estimated using MTD 11-34 approach. The deformation behavior of the adjacent span of in-span hinges is currently being studied to determine its impact on the hinge curl.

CONCLUSIONS

Based on this study, it can be concluded that:

1. Despite the variation in the bridge geometry and prestress forces, all three bridges exhibit the same trend in the upward movement of the short cantilevers adjacent to in-span hinges.
2. The measured instantaneous hinge curls are considerably higher than those obtained from the current design equations shown in MTD 11-34. The ratio of measured to estimated movements ranged from 2.2 to 6.3.
3. The underestimation of hinge curl appears to be due to the rotation of cap beam and movement in the adjacent span, both of which are currently ignored in the design equations.

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